APPENDIX B

GEOTECHNICAL INVESTIGATION REPORTING AND GRADING PLAN REVIEW CANYON CROSSING AT CANYON SPRINGS PROJECT

JANUARY 15, 2004

CONVERSE CONSULTANTS

January 15, 2004

Mr. Jeff Adams
Transcan Development, LLC
3189 Danville Blvd., Suite 245
Alamo, California 94507

Subject: **GEOTECHNICAL INVESTIGATION REPORT AND GRADING PLAN REVIEW**
Canyon Crossing at Canyon Springs Project
Southeast Corner of State Highway 60 and Interstate 215
Riverside County, California
Converse Project No. 03-81-269-01

Dear Mr. Adams:

Converse Consultants (Converse) has prepared this *Geotechnical Investigation Report and Grading Plan Review* to present the findings of our geotechnical exploration performed for the proposed commercial development located at the southeast corner of the intersection of State Highway 60 and Interstate 215 in Riverside County, California. This report was prepared in accordance with our proposal dated September 8, 2003 and your authorization dated October 15, 2003.

It is our opinion that the subject site can be adequately developed from a geotechnical standpoint to support the proposed commercial development, provided the findings, conclusions, and recommendations presented in this report are incorporated in the preparation of the final grading plan, foundation design, and construction of the project.

The geotechnical recommendations provided in this report are for grading, and preliminary foundation and slab design. An *As-Built Geologic And Soil Compaction Report* should be prepared at the completion of grading, which will include final foundation and slab design recommendations.

We appreciate this opportunity to be of continued service to Transcan Development, LLC. If you should have any questions, please feel free to contact the undersigned at (909) 796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, Ph.D., G. E.
COB/Regional Manager

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PROFESSIONAL CERTIFICATION

This report has been prepared by the staff of Converse Consultants under the professional supervision of the registered engineer and certified engineering geologist(s) whose seals and signatures appear hereon.

The findings, recommendations and professional opinions presented in this report were prepared in accordance with generally accepted professional geological and engineering practice at this time in Southern California. There is no other warranty, either expressed or implied.

Roy J. Rushing, C.E.G.
Senior Geologist

Hashmi S. E. Quazi, Ph.D., G.E.
COB/Regional Manager

EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions, and recommendations as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The approximately 54 acres of undeveloped land is located at the southeast corner of the intersection of State Highway 60 and Interstate 215 in Riverside County, California. Retail shopping centers exist south and east of the site. No structures or development exist on the site; however, easements for the California Aqueduct and Metropolitan Water Department cross the eastern and southern portions of the site, respectively, intersecting near the existing Wal-Mart store to the east.

- The subject site is to be developed for one- and/or two-story commercial structures, which are likely to be of masonry block or concrete tilt-up construction. It is our understanding that the development will include nine (9) structures and associated truck driveways with curbs and gutters, parking areas, sidewalks, landscaped areas, and under- and above-ground utilities.

- Our scope of work included the following tasks: site reconnaissance, subsurface exploration using hollow-stem auger borings, laboratory testing, engineering analyses, and preparation of this report.

- Twenty (20) exploratory borings were drilled within the project site to depths ranging from three (3) to twenty-eight (28) feet below existing ground surface (bgs).

- There are no active faults projecting toward or crossing the site. The site is not situated within a currently designated State of California Earthquake Fault Zone. The site is, however, located in a seismically active area. Ground shaking from earthquakes associated with nearby and distant faults may occur during the lifetime of the project. In accordance with the Uniform Building Code (UBC, 1997), Table 16-J; the geologic subgrade classification is S_C. The seismic design coefficients at the site in accordance with Tables 16-Q through 16-T are:

$$C_a=0.40 \qquad N_a=1.0$$
$$C_v=0.58 \qquad N_v=1.0$$

- Based on available information, the depth to groundwater within the general vicinity of the site is more than 50 feet bgs. In addition, the site is underlain by granitic bedrock,

in which groundwater typically flows along deep fractures. Therefore, groundwater need not be considered in design and construction.

- The liquefaction hazard during earthquake ground shaking at the site is essentially nil due to the presence of shallow bedrock across the site.

- The potential for occurrence of seismic hazards including surface fault rupture, seismically induced differential ground settlement, slope instability, lateral spreading, and landslides are considered to be very low to low. Based on the site location, the potential for earthquake induced flooding does not exist.

- The project site is underlain by residual soil and colluvial deposits and granitic bedrock to the maximum explored depth 28 feet bgs. The thickness of residual soil/colluvium ranges from approximately 3 feet to 13.5 feet across most of the site. The residual soil/colluvial deposits generally consist of silty and clayey sand with variable amounts of gravel. Relatively shallow granitic bedrock was encountered in all of the borings beneath the soil/colluvium.

- Boring logs indicate that difficult excavation should be anticipated at the site. Refusal with the hollow-stem auger drill occurred at depths between three (3) and twenty-eight (28) feet bgs, primarily in the northwest corner of the project. Based upon the grading plan, the proposed finish grade elevation for the proposed pads will require up to fourteen (14) feet of fill placed in this area.

- Laboratory testing consisting of moisture-density, expansion index, compaction, direct shear, swell/collapse, R-value, and corrosivity tests were performed for soil classification purposes and evaluation of relevant physical characteristics and engineering properties.

- The majority of the site will require grubbing and removal of scattered debris, weeds, and other unsuitable materials prior to commencement of grading.

- Any undocumented fill soils should be removed and replaced as compacted fill.

- In general, the residual/colluvial soils that overlie the bedrock at the site are not considered suitable for supporting structures or additional fill. Site grading should include removal and re-compaction of these unsuitable materials to a maximum depth of five (5) feet below the bottom of footings.

- Subgrade soil surfaces that will receive compacted fill shall be scarified to a depth of at least 12 inches. The scarified material to be moisture-conditioned to within two (2) percent above optimum moisture content. Scarified soil shall then be compacted to at least 90 percent relative compaction.

- Cut/fill transition pads should be over-excavated to provide at least five (5) feet of compacted fill over the entire pad.

- Cut and fill slopes should not be steeper than 2:1 horizontal:vertical (H:V). Based on our evaluation of the stability, cut and fill slopes constructed at a gradient of 2:1 (H:V) should have an adequate safety factor.

- Based on the materials encountered in the exploratory borings, temporary excavations may be constructed vertical up to an excavation depth of five (5) feet bgs and not steeper than 1:1 (H:V) for an excavation from five (5) to ten (10) feet deep.

- All trench backfill should be compacted to a minimum relative compaction of 90 percent as per ASTM Standard D1557 test method. At least the upper 12 inches of trench underlying pavements should be compacted to a minimum of 95 percent relative compaction.

- For footings founded on compacted fill, a minimum of one footing width of compacted fill should be provided beneath the bottom of the footings.

- Commercial one-story masonry block or concrete tilt-up walls, lightly loaded structures may be supported on conventional continuous and/or spread footings. Interior and exterior footings may be designed based on an allowable net bearing capacity of 3,000 pounds per square foot (psf).

- Cantilevered earth retaining walls should be designed based on an active earth pressure equal to that developed by a fluid of density of 40 pounds per cubic foot (pcf). These pressures assume a level ground surface behind the wall for a distance greater than the wall height. If water pressure is allowed to build up behind the walls, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the wall. At-rest earth pressure can be taken as equal to that developed by a fluid density of 60 pcf.

- Resistance to lateral loads may be provided by the passive earth pressures and frictional resistance at the base of the footing. A coefficient of friction of 0.40 between concrete and soil may be used with the dead load forces. Passive earth pressure of 250 psf per foot of footing depth may be used. The passive resistance should be limited to a maximum of 3,000 psf.

Based on our investigation, the project site can be developed to support the proposed residential development, provided the findings and conclusions presented in this geotechnical investigation report are incorporated in the planning, design and construction of the project.

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1.0 INTRODUCTION

This report presents the findings of our geotechnical investigation performed at the site of the proposed commercial development. The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils and bedrock materials; and to provide preliminary recommendations regarding general site grading and for design and construction of foundations for commercial structures and various facilities generally associated with such development.

The approximately 54-acre site is located southeast of the intersection of State Highway 60 and Interstate 215 in Riverside County, California. The site location is shown in Figure No. 1, *Site Location Map.*

A tentative map labeled "Study No. 2", dated May 15, 2003, prepared by Hunsaker and Associates, Inc., was originally used to locate the test borings. After completion of the field exploration, a ________________________, dated ________ prepared by Hansaker and Associates, was used as a basis for the attached Drawing No. 1, *Approximate Boring Location Map,* and for development of grading and design recommendations.

This report is written for the project described herein and is intended for use solely by Transcan Development, LLC and its design team. It should not be used as a bidding document, but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

Our recommendations provided in this report did not include interpretations of environmental impacts and assumes no environmental concerns for the site.

2.0 PROJECT DESCRIPTION

2.1 Project Location and Boundaries

The approximately 54-acre L-shaped parcel is a gentle, rolling plain with minor natural swales. The northern boundary of the project is State Highway 60. The western boundary is Interstate Highway 215. Residential developments and retail districts exist south and east of the site. No structures or development exist on the site surface; however, easements for the California Aqueduct and Metropolitan Water Department cross the eastern and southern portions of the site, respectively, intersecting near the existing Wal-Mart store. Native grasses and scattered trees exist on-site. Drainage on-site is by sheetflow and by minor swales to the west and southwest. The following photograph shows the typical surface conditions.

Looking northwest across site.

2.2 Proposed Development

Eight (8) one-story commercial structures are planned for construction on the site. It is our understanding that the development will also include construction of associated truck driveways with curbs and gutters, sidewalks, parking areas and above and underground utilities. The structures are likely to be of masonry block or concrete tilt-up construction, founded on continuous and/or isolated footing foundations with slab-on-grade.

Presently, the project will not include any retaining walls of significant height. A fill slope is currently proposed parallel to Interstate 215 along the western property boundary. As shown on Drawing No. 1, *Approximate Boring Location Map*, part of the northern area will be within a CalTrans right-of-way for a future transition lane. A significant cut slope, approximately 30 feet high, will be constructed along the southern flank of the hill and a west-facing slope between the north-south trending California Aqueduct easement and the proposed Major F structure and parking (see Drawing No. 1 *Geology and Approximate Boring Location Map).*

3.0 SCOPE OF WORK

The scope of our present investigation included a review of existing information, site reconnaissance, subsurface exploration, laboratory testing, engineering and geological analyses, and preparation of this report. The scope of work included the following tasks in the subsections below.

3.1 Literature Review

As part of this investigation, we have reviewed available and pertinent literature, reports and maps described in the *References*, included at the end of this report.

3.2 Field Exploration

Our field exploration included a site reconnaissance by a Converse geologist. The purpose of the site reconnaissance was to observe surface conditions and to select exploratory boring locations. The approximate locations of the exploratory borings are shown on Drawing No. 1, *Geology and Approximate Boring Location Map.*

A total of 20 exploratory borings were drilled within the project site. The borings were drilled to a maximum depth of 28 feet below the ground surface (bgs) using truck-mounted rig equipped with an 8-inch diameter hollow-stem auger. The actual depths drilled were determined by the previously proposed development at the specific location and the presence of hard bedrock.

The subsurface conditions encountered in the exploratory borings were visually logged a Converse geologist. Relatively undisturbed ring and bulk samples of the representative subsurface materials were obtained from the borings at frequent depth intervals for the purpose of field identification and laboratory testing. One standard penetration test was attempted with very limited success.

A more detailed description of the field exploration procedures and Logs of Borings are presented in Appendix A, *Field Exploration.*

3.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in the soils classification and to evaluate relevant engineering properties of the site soils. These tests included:

- *In situ* moisture contents and dry densities (ASTM Standard D2216)
- Expansion Potential (CBC-1998)
- Maximum dry density and optimum-moisture content relationship (ASTM Standard D1557)
- Direct shear (ASTM Standard D3080)
- R-Value (ASTM D2844)
- Swell/Collapse (ASTM Standard D2435/D5333)

- Soil corrosivity tests (ASTM Standards D512, D516, D513, D1125, D1126, D2791, G51, and G57)

For a description of the laboratory test methods and test results see Appendix B, *Laboratory Testing Program.* For *in situ* moisture and density data see the Logs of Exploratory Borings in Appendix A, *Field Exploration.*

3.4 Report Preparation

Data obtained from the field exploration and laboratory testing program were evaluated. Geotechnical analyses were performed and this report was prepared to present our findings, conclusions, and recommendations for the proposed residential development.

4.0 FINDINGS

A general description of the site conditions including existing surface conditions, site geology, and subsurface conditions encountered during our field exploration are presented in the following subsections. A discussion of engineering characteristics of the subsurface materials is also presented.

4.1 Site Conditions

The existing elevations of the site varies from approximately 1,540 to 1,580 feet above Mean Sea Level (MSL), decreasing in elevation from north to south. A large hill occupies the area adjacent to the northwest corner of the site. The majority of the development will occur on the gently sloping portion of the site.

4.2 Existing Surface Conditions

The soils encountered at the surface consisted of fine-grained silty and clayey sand with minor amounts of gravel. These materials were found to be dry and often porous.

At the time of our field investigation, vegetation consisted of a thick covering of dry grasses covering essentially the entire site, with the exception of several dirt roads.

4.3 Geologic Setting

The subject commercial development is located in the Peninsular Ranges geomorphic province of Southern California. The Peninsular Ranges province is a physiographic region that extends from the San Gabriel and San Bernardino Mountains in the north into Baja California in the south. The province is characterized by northwest-trending mountain ranges and valleys, and similarly oriented faults.

4.4 Site Geology

The site is located on the southern flank of a west to east trending ridge. Geologic units underlying the site include Cretaceous period granitic bedrock, with a layer of residual/colluvial soils, derived from the in-place weathering of the underlying bedrock.

4.5 Subsurface Conditions

4.5.1 Subsurface Profile

Colluvial soils and granitic bedrock underlie the project site as shown in the Logs of Borings, included in Appendix A, *Field Exploration*. The residual/colluvial soils generally consist of silty and clayey sand with minor amounts of gravel. The bedrock was found to consist primarily of granite weathered primarily to silty sand. For additional information on the subsurface conditions, see Appendix A, *Field Exploration*.

In situ dry densities of the upper ten (10) feet of the site residual/colluvial soils range from 99 to 126 pounds per cubic foot (pcf). At the time of our field investigation, the moisture content of the upper ten (10) feet the site soils ranged from 3 to 14 percent. The underlying bedrock, where found in the upper ten (10) feet bgs, ranged from 100 to 121 pcf and four (4) to fifteen (15) percent moisture. The laboratory results of the *in situ* moisture content and dry densities are presented in the Logs of Borings in Appendix A, *Field Exploration*.

An expansion index was performed on a sample of silty sand with a trace to little clay, representative of the deeply weathered residual soils found in some areas of the site. This soil was found to have an Expansion Index (EI) of 37. Based on this test result, these soils have a low expansion potential (see Table No. B-1, *Results of Expansion Test Results,* in Appendix B, *Laboratory Testing Program.*

Typical moisture/density relationships of a representative near surface soil sample are presented in Drawing No. B-1, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* Based on the above results, the maximum dry density of the near surface soils is 132.5 pounds per cubic foot (pcf) with an optimum moisture content of 8.5 percent.

The results of a direct shear test performed on relatively undisturbed samples of the site soils are presented in Drawing No. B-2, *Direct Shear Test Results*, included in Appendix B, *Laboratory Testing Program* and on Table No. B-2, *Summary of Direct Shear Test Results* in Appendix B, *Laboratory Testing Program.* The laboratory friction angle and cohesion intercept of the silty sand tested was 34 degrees and 350 pounds per square foot (psf), respectively. Shearing strength of the site soils can be considered to be in the moderate range.

Two (2) R-value tests were performed on representative bulk soil samples. The test results are shown in Table No. B-3, *Summary of R-value Test Results*, in Appendix B, *Laboratory Testing Program*. Based on the test results, the R-value of near surface site soils ranges were 36 and 40.

The collapse potential under a vertical stress of 2.0 kips-per-square-foot (ksf) was performed on three (3) relatively undisturbed representative samples collected at five (5) feet bgs. Percent collapse values ranged from 0.2 to 3.4 percent. This range of values corresponds to low to moderate collapse potential (1%-5%) based on the NAVFAC Design Manual 7.1. Test results are presented in Table No. 4, *Summary of Collapse Test Results*, in Appendix B, *Laboratory Testing Program*.

4.5.2 Groundwater

At the time of our field investigation, groundwater was not encountered in any of the exploratory borings drilled to a maximum of twenty-eight (28) feet bgs. Based on information obtained from the Fall 2002 Cooperative Well Measuring Program Report, produced by Western Municipal Water District, the depth to groundwater in several wells located about one and one-half miles to the east/southeast, was approximately 75 feet bgs in April of 2002. Typically in granitic bedrock, groundwater is found flowing along fractures that can be within a few tens of feet from the surface, or deeper. Based on the subsurface materials and the expected groundwater conditions, it is our opinion that groundwater will not have any adverse effect on the proposed project and need not be considered in the design and construction. During grading all cut slopes will be observed by an engineering geologist for occurrence of unexpected groundwater flowing along fractures. Any mitigation requirements will be provided at that time.

4.6 Excavatability

Based on the results of our field exploration, excavation of the bedrock materials at the site is anticipated to present some difficulty, especially in deeper excavations such as for the cut slope along the northern edge of the site and along the western side of the California Aqueduct alignment. Excavation of the residual/colluvial soils is expected to be easily accomplished with conventional heavy duty grading equipment.

4.7 Soil Corrosivity

M.J. Schiff and Associates of Claremont, California tested a representative site soil sample. The testing included minimum resistivity, pH, soluble sulfates, and chloride content. The results are presented in Table No. B-5, *Soil Corrosivity Test Results,* included in Appendix B, *Laboratory Testing Program,* and is discussed below.

The sulfate content of the sample tested was "not detected", which indicates that site soils are not deleterious to concrete. Type II Portland cement may be used for the construction of the foundations (See Table 19-A-4 of the UBC).

The chloride content in this sample was also found to be "not detected", which is not significant in evaluating soil corrosivity.

The pH value was 7.4. Within this range of pH, the soils should not affect common construction materials.

The measured value of the minimum electrical resistivity when saturated was 8,000 ohm-cm. These values indicate that site soil is moderately corrosive to ferrous metals. The test results are presented in Appendix B, *Laboratory Testing Program.*

If necessary, a corrosion engineer should be consulted to provide mitigation recommendations. However, some general, conventional corrosion mitigation measures include the following:

- All steel reinforcement should have at least three (3) inches of concrete cover where cast against soil, unformed.

- As a minimum, below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel or Portland cement mortar.

4.8 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the test pit locations.

5.0 FAULTING AND SEISMICITY

5.1 Faulting

Based on our review of the available information, there are no known active faults projecting toward or extending across the project site. An active fault is defined as the one that has had surface displacement within Holocene time (about the last 11,000 years). The site is not situated within a currently designated State of California Earthquake Fault Zone.

5.2 *Seismicity*

The project site is situated in a seismically active region. As is the case for most areas of Southern California, ground shaking resulting from earthquakes associated with nearby and distant faults may occur. During the life of the project, seismic activity associated with active faults in the area may generate moderate to strong ground shaking at the site.

According to the Uniform Building Code (1997), the project site is situated in Seismic Zone 4. Major damage corresponding to intensities VIII or higher on the Modified Mercalli Intensity Scale may occur within this zone. Seismic Zone 4 also includes those areas that lie within a zone of major (Richter Magnitude, M>7) historic earthquakes and recent high levels of seismicity.

Various active faults within a distance of 100-km (62-mile) from the site and capable of generating significant ground motion (>0.10g, where g is the acceleration due to gravity) are listed in Table No. 1, *Ground Acceleration From Nearby Active Faults*. The maximum peak horizontal site acceleration presented in Table No. 1, *Ground Acceleration From Nearby Active Faults,* for each fault is from the Maximum Credible Earthquake (MCE) that the particular fault is capable of generating.

MCE is defined as the maximum seismic event that a particular fault is theoretically capable of producing and is evaluated based upon existing geologic and seismologic evidence. Various researchers (Wesnousky, 1986) have presented estimated values of MCE for various California faults. Probabilistic ground acceleration at a given site corresponding to the MCE of a given fault may be estimated by utilizing various attenuation relationships published in the literature. Such an analysis was performed by utilizing the computer program FRISKSP developed by Blake (2000) for various active faults within a 100-km (62-mile) radius from the project site. The MCE's assigned to various faults are those selected by Blake.

An analysis was performed to evaluate seismically induced ground motions at the site associated with different risk levels. Such analysis involves selection of various design-level earthquakes associated with different degrees of risk. Available seismic data and various published attenuation relationships are then utilized to determine various seismic parameters associated with the nearby known active faults.

A probabilistic seismic analysis calculates the probable ground accelerations for various return periods as a function of distance of the site from a particular fault. Probability of exceedence versus site-specific ground acceleration determined by using FRISKSP computer program as shown in Figure No. 2, *Probability of Exceedence*. Based on these results, a maximum horizontal peak ground acceleration of 0.60g at the proposed site has a 10 percent probability of being exceeded in 50 years. For design purposes, the vertical acceleration may be taken as two-thirds of the horizontal acceleration.

The results of the analysis are presented in Table No. 1, *Seismic Characteristic of Nearby Active Faults*. The peak horizontal site accelerations presented in the following table are obtained in accordance with the attenuation relationship suggested by Campbell and Bozorgia (1997 Rev.).

Table No. 1, Seismic Characteristics of Nearby Active Faults

FAULT NAME	APPROX. DISTANCE TO FAULT FROM PROJECT SITE (miles)	ASSIGNED MAXIMUM CREDIBLE EARTHQUAKE MAGNITUDE M_w[1]	MAXIMUM PEAK HORIZONTAL GROUND ACCELERATION AT PROJECT SITE (g)	SLIP RATE (MM/YR)	FAULT TYPE[2]
San Jacinto – San Bernardino	6.0	6.7	0.55	12.0	SS
San Jacinto – San Jacinto Valley	6.2	6.9	0.58	12.0	SS
San Andreas – San Bernardino	15.7	7.4	0.31	24.0	SS
San Andreas – Southern	15.7	7.4	0.33	24.0	SS
Elsinore – Glen Ivy	17.8	6.8	0.20	5.0	SS
Chino-Central Ave. (Elsinore)	18.9	6.7	0.19	1.0	DS
Cucamonga	20.7	7.0	0.21	5.0	DS
Elsinore – Temecula	20.9	6.8	0.17	5.0	SS
Whittier	21.0	6.8	0.17	2.5	SS
North Frontal Fault Zone (West)	22.9	7.0	0.18	1.0	DS
Cleghorn	23.2	6.5	0.12	3.0	SS

Notes:

1 Moment Magnitude M_w of earthquake expected for rupture of entire fault length, estimated with slip-rate dependent empirical relations between seismic moment M_o and fault length and assuming the empirical relationship Log M_o=1.5 M_w + 16.1 (Hanks and Kanamori, 1979).

2 SS = Strike Slip; DS = Dip Slip

5.4 UBC Seismic Design Parameters

In accordance with the Uniform Building Code (UBC, 1997) Table 16-J; the geologic subgrade classification is S_c. The seismic design coefficients for the site in accordance with Tables 16-Q through 16-T are:

C_a=0.40 N_a=1.0
C_v=0.58 N_v=1.0

5.5 Other Effects of Seismic Events

Aside from generating damaging ground motion, a nearby seismic event may impact a project by inducing landslides, earthquake-induced flooding, tsunamis, seiches, soil liquefaction, differential settlement and ground lurching. A site-specific discussion on each of the above secondary effects is provided below:

Landslides: Seismically induced landslides and slope failures are common occurrences during or soon after large earthquakes. The existing slopes at the site are underlain by high-strength bedrock materials. The existing natural slopes have experienced numerous large earthquakes in the past, and there is no evidence of previous slope failures at the site. Therefore, seismically induced landsliding of the natural slopes at the site appears unlikely.
The only significant fill slope currently proposed will be adjacent to the western property boundary, descending to the I-15 right-of-way. Fill slopes constructed at an inclination of 2:1 (H:V) are expected to be seismically stable. Because of the high strength of the on-site bedrock materials, the proposed cut slopes are also anticipated to be seismically stable at the currently proposed 2:1 (H:V) inclination. See Section 6.0, *Slope Stability*, for additional discussion.

Earthquake-Induced Flooding: This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Review of the area adjacent to the site indicates that there are no significant up-gradient lakes or reservoirs with the potential of flooding the site.

Tsunamis: Tsunamis are tidal waves generated by fault displacement or major ground movement. Based on the inland location of the site, tsunamis do not pose a hazard to the site.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Review of the area adjacent to the site indicates that there are no significant up-gradient lakes or reservoirs with the potential of flooding the site.

Soil Liquefaction: Liquefaction is defined as the phenomenon in a soil mass due to the development of excess pore pressures, which a soil mass suffers a substantial reduction in its shear strength. During earthquakes, excess pore pressures in saturated sandy soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction. Any susceptible soils will be removed during grading operations and replaced with compacted fill. The site is underlain by non-liquefiable bedrock. Therefore, the site is not susceptible to liquefaction.

Differential Settlement and Ground Lurching: The potential for significant differential settlement and ground lurching during earthquakes is considered to be very low to low at the site because of the presence of uncompressible bedrock at relatively shallow depths across the site. Differential settlement of fill soils can be minimized by adherence to the site preparation and grading recommendations contained in subsequent sections of this report.

6.0 SLOPE STABILITY

The proposed grading plan indicates that cut slopes up to a maximum 30 feet high are currently proposed. The only currently proposed fill slope will be along the western property line descending to the I-15 right-of-way. All slopes are proposed at an inclination of 2:1 (H:V).

Stability of rock slopes is highly dependent on the presence and orientation of any planar discontinuities such as bedding, fractures, faults, etc. For failure to occur along planar discontinuities, the discontinuity (or the intersection between discontinuities) must dip out-of-slope at an angle flatter than the slope face (so that it daylights in the slope), and at an angle steeper than the shear strength along the discontinuity. This is unlikely for 2:1 (H:V) slopes because it would require the presence of very weak planar discontinuities, which appears unlikely in the bedrock which underlie the site, as observed in our exploratory borings.

In the absence of significant adversely oriented structural discontinuities, the proposed cut slopes are anticipated to be grossly stable because of the high strength of the on-site bedrock, as observed in the exploratory borings. Geologic observation and mapping of the cut slopes should be performed during grading to confirm this finding.

The proposed fill slope will be constricted from on-site residual/colluvial soils and excavated bedrock. This material will be grossly stable at the proposed 2:1 (H:V) inclination.

7.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, the project site is suitable for the proposed commercial development, provided that the recommendations presented in this report are incorporated in preparation of the grading plan, foundation design, and construction of the project.

An *As-Built Geologic And Soil Compaction Report* should be prepared at the completion of site grading to provide final foundation and slab design recommendations.

8.0 EARTHWORK/SITE GRADING RECOMMENDATIONS

8.1 General

This section contains our general recommendations regarding earthwork for the proposed development. These recommendations are based on the results of our field exploration, laboratory testing, and data evaluation as presented in the preceding sections as the date applies to the _______________ grading plan. These recommendations may need to be modified based on observation of the actual field conditions during grading.

Site preparation will require removal of any fill, stockpiles, weeds/vegetation, debris, and organic and non-organic materials resulting from the cleaning and grubbing operation and that material should be hauled off the site. Removal of localized areas deeper than those

documented may be required during grading. Therefore, some variations in the depth of over-excavation recommended in this report should be anticipated.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill and/or structures. All fill should be placed on competent native materials as determined by Converse representatives and in accordance with the specifications presented in this section and in Appendix C, *Recommended Earthwork Specifications.*

Prior to compaction, fill material should be thoroughly mixed and moisture conditioned, when necessary, within three (3) percent above optimum for granular fill soils, and within two (2) percent or above of the optimum moisture content for fine-grained fill soils. All fill, if not specified otherwise in this report, should be compacted to at least 90 percent relative compaction in accordance with the ASTM Standard D1557 test method.

The colluvial soils are expected to be an essentially fine- to coarse-grained material. The bedrock material could, depending upon ease of excavation, end up with occasional rocks up to and larger than two (2) feet in diameter. The upper three (3) feet of fill below finish pad elevations should not contain rock larger than six (6) inches in maximum dimension.

The footings of buildings and any retaining walls should be setback as presented in Figure No. 18-I-1 of the Uniform Building Code (2001, edition).

Cross sections across a few of the structures showing the presence of the colluvium and bedrock, transitions between cut and fill and colluvium and bedrock are presented on Drawing No. 2, *Geologic Cross Sections.* The approximate locations of the sections are shown on Drawing No. 1, *Geology and Approximate Boring Location Map*

8.2 Overexcavation/Removal

Actual removal depths will depend on observation of the site conditions encountered during grading. In general, the surficial residual/colluvial soils are loose and unsuitable for the support of structures or additional fill soils. Therefore, these soils should be removed to bedrock, processed and replaced as compacted fill in all structural areas and in areas to receive fill. In most areas of the site, the residual/colluvial soils were found to range from less than 3 feet to about 13.5 feet thick. At least the upper 12 inches of weathered bedrock should also be removed prior to processing and placement of compacted fill.

In order to provide uniform foundation support and to facilitate excavations for foundations and utilities, all fill and transition lots should be over-excavated to a depth of at least five (5) feet below finished grade. This over-excavation should extend laterally at least five (5) feet or equal to the depth of over-excavation, whichever is greater, outside the exterior footing lines. Because of the removal of the colluvial soils and weathered bedrock, as discussed above, competent bedrock should be encountered in the base of all over-excavations. Therefore, minimal preparation of the over-excavation bottoms should be required.

As an alterative to the over-excavation of the cut portion of transition pads, deepening footings through the fill portion into competent bedrock may be considered.

8.3 Permanent Cut Slopes

Based on our preliminary evaluation, cut slopes in the encountered materials up to 30 feet in height and no steeper than 2:1 (H:V) should be stable. Structures should be set back from top or bottom of cut slopes as shown on Figure 16-I-1 of the 2001 Edition of the UBC and designed according to Section 33.5 of Chapter 33 of the UBC.

Geologic observation of all cut slopes should be conducted during grading to observe, if any, adversely oriented planes of weakness (i.e., fractures or joints) are present. If adverse conditions are found during grading, buttress or stabilization fill may be required.

8.4 Permanent Fill slopes

Fill slopes should be constructed with slope ratios no steeper than 2:1 (H:V). Fill slopes in excess of 15 feet in height are not anticipated. Fill slopes should be properly compacted out to the slope face. This may be achieved by either overbuilding and cutting back to the compacted core or by utilizing other methods that meet the intent of the project specifications. If fill is placed over cut slopes or natural slopes, the fill should be benched into competent alluvium or firm bedrock as shown on Figure No. C-1, *Fill Over Natural*

Slope, in Appendix C, *Recommended Earthwork Specifications*.

Fill slopes, properly compacted to the slope-face, graded no steeper than 2:1 (H:V), and are not higher than 15 feet, should be grossly stable.

Permanent structures should be set back from graded slopes in accordance with Chapter 18 (CBC Figure 18-I-1) of the California Building Code (2001 Edition).

8.5 Slope Protection and Maintenance

Slopes should be planted as soon as possible after construction. Slopes will require maintenance through time to perform in a satisfactory manner. In most cases, lot and site maintenance can be performed along with normal care of the grounds and landscaping. The cost of maintenance is less expensive than repair resulting from neglect.

Most hillside slope stability problems are associated with water. Uncontrolled water from a broken pipe, excess landscape watering, or exceptionally wet weather causes the most damage. Drainage and erosion control are important aspects of slope stability, and the provisions incorporated into the graded site must not be altered without competent professional advice.

Any terrace drains and brow ditches on the slopes should be periodically maintained and kept clear so that water will not overflow onto the slope, causing erosion. All subdrains should be kept open and clear of debris and soil that could block them. Landscaping on the slopes should disturb the soil as little as possible and utilize drought-resistant plants that require a minimum amount of irrigation. Wet spots on or around the site should be noted and brought to the attention of Converse or an experienced geotechnical engineer. These may be natural seeps or an indication of broken water or sewer lines.

Watering should be limited or stopped altogether during the rainy season when little irrigation is required. Slopes should not be over-irrigated. Ground cover and other vegetation will require moisture during the hot summer months, but during the wet season, irrigation can cause ground cover to pull loose. This not only destroys the cover but also starts serious erosion and or surficial instability. It is suggested to consult a professional landscape architect for planting and irrigation recommendations.

8.6 Temporary Sloped Excavations

Based on the materials encountered in the exploratory borings, sloped temporary excavations may be constructed according to the slope ratios presented in Table No. 2, *Slope Ratios for Temporary Excavation*. Temporary cuts encountering loose fill, or loose and dry sand may have to be constructed at a flatter gradient than presented in the following table.

Table No. 2, Slope Ratios for Temporary Excavation

Maximum Depth Of Cut (feet)	Maximum Slope Ratio* (horizontal:vertical)
0 – 5	Vertical (in compacted fill or bedrock)
5 – 10	1:1 (in bedrock)

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction equipment, should not be placed within five (5) feet of the unsupported trench edge. The above maximum slopes are based on a maximum height of six (6) feet of stockpiled soils placed at least five (5) feet from the trench edge.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be followed. The soils exposed in cuts should be observed during excavation by a Converse representative. Modifications of slope ratios for temporary cuts may be required if potentially differing and/or unstable soil conditions are encountered.

Temporary shoring is not anticipated for this project. If shoring is required, this office should be contacted for earth pressure recommendations.

8.7 Shrinkage and Subsidence

The shrinkage and/or bulking would depend on, among other factors, the depth of cut and/or fill, and the grading method and equipment utilized. For preliminary estimate purposes, bulking and shrinkage factors for various units of earth material at the site may be taken as presented below:

- The shrinkage factor for the colluvial soils is estimated to be in the range of 10 to 25 percent.
- Subsidence would depend on the construction methods, including type of equipment utilized. For estimation purposes, ground subsidence may be taken as 0.5-inch.

Bulking of the on-site bedrock materials should be anticipated. The bulking percentage is difficult to estimate, however a range of 3 to 8 percent may be considered for planning purposes.

Although these shrinkage and bulking values are only an approximation, they represent our best estimates of the factors to be used to calculate volume changes that may occur during grading. If more accurate shrinkage/bulking factors are needed, it is recommended that field testing using the actual equipment and grading techniques be conducted.

8.8 Site Drainage

Adequate positive drainage should be provided away from building pad areas to prevent ponding and to reduce percolation of water into the foundation materials. There should be a gradient of at least two (2) percent away from foundations. Planters and landscaped areas adjacent to the buildings should be designed and irrigated to minimize water infiltration into the subgrade soils.

If irrigation waters are excessive, they can percolate to subsurface soils. Such subsurface water will flow from raised-grade pads to adjacent lower-grade pads. This will supplement the percolating subsurface water resulting from the irrigation adjacent to the lower pads and can result in overly saturated and/or perched groundwater conditions. Irrigation should be maintained such that it does not result in excess groundwater. Slopes adjacent to raised pads should be provided with a subdrain along a toe-of-fill keyway to intercept subsurface water flow.

Surface drainage should preclude the possibility of flow over slope faces with the use of brow ditches, earth berms, and other methods.

Adequate drainage should be provided for any cut and/or fill slopes, landscaped areas outside building pads. A desirable drainage gradient is one (1) percent for paved areas and two (2) percent in landscaped areas.

Gutters and downspouts should be installed on roofs, and runoff should be directed to the street or controlled drainage outlets through non-erosive devices. Surface drainage should be directed to suitable non-erosive devices. Slope drainage should be constructed in accordance with the California Building Code (2001).

9.0 BURIED UTILITIES

9.1 General

Buried utility construction will involve open-cut trench excavation, pipe subgrade preparation, and placement of bedding, if used, before placing the pipe. The trench will then be backfilled with compacted fill to final grade.

The backfill material for the pipe zone is controlled by local codes and should be selected by the pipe designer. The pipe zone is defined as the portion of the trench section extending from the pipe invert to one foot above the top of the pipe. The remainder of the trench section above the pipe zone is defined as the trench zone.

Where the conduit underlies slab-on-grade, the trench backfill should be placed and compacted in accordance with the specifications set forth in Section 9.3, *Recommended Specifications for Placement of Trench Backfill.* Excavated on-site soils free of organic matter may be used to backfill the trench zone. Imported trench backfill should be approved by Converse prior to delivery.

9.2 Pipe Bedding

Pipe bedding is defined as the material supporting the pipe. To provide uniform and firm support for the pipeline, free-draining granular soil should be used as pipe bedding material. For flexible pipes, excavated sandy materials may be used as bedding material. Crushed rock or gravel may be used for rigid pipes. The thickness and specifications of the bedding materials under the pipe, if any, should be selected by the pipeline design engineer.

We recommend that granular material with a sand equivalent (SE) greater than 30 be used for bedding material. Bedding material for the pipes should be free from oversized particles (greater than 1 inch). Migration of fines from the surrounding native and/or fill soils must be considered in selecting the gradation of the imported bedding materials. We recommend that the following gradation criteria should be met:

$$D_{15} < 0.1 \text{ inch and } D_{50} < 0.75 \text{ inch}$$

Where D_{15} and D_{50} represent particle sizes of the bedding material corresponding to 15 percent and 50 percent passing by weight, respectively. The maximum size of bedding material should not exceed one inch.

Specifications for bedding including required backfill requirements surrounding the pipe should be specified by the pipe manufacturer.

9.3 Recommended Specifications for Placement of Trench Backfill

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

- All trench backfill should be compacted to a minimum relative compaction of 90 percent as per the ASTM Standard D1557 test method. At least the upper 12 inches of trench underlying pavements and structures such as concrete slabs-on-grade shall be compacted to not less than 95 percent of the maximum dry density.

- Compaction of trench backfill in off-site street improvements should be per Caltrans specifications for field density tests results and maximum density – optimum moisture determinations.

- Rocks larger than one inch in diameter should not be placed within 12 inches of the top of the pipeline or within the upper 12 inches of pavement or structure subgrade. No more than 30 percent of the backfill volume shall be larger than 3/4-inch in the largest dimension and rocks shall be well mixed with finer soil.
- Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers to achieve the density specified herein. The backfill materials shall be brought to, within two (2) percent above optimum moisture content, then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer shall be evenly s spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

- The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

- Where applicable the field density of the compacted soil shall be measured in accordance with the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.

- Observation and field tests should be performed by Converse during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive efforts shall be made with the adjustment of moisture content, as necessary, until the specified compaction is obtained.

- It should be the responsibility of the contractor to maintain safe conditions during cut and/or fill operations.

- Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

10.0 DESIGN AND CONSTRUCTION RECOMMENDATIONS

10.1 Foundation and Retaining Wall Design Parameters

The recommended design parameters are preliminary. The final bearing pressures and lateral resistances will be provided at the completion of grading. Such recommendations would be based on testing and evaluation of the soil conditions under the proposed foundation levels after the completion of rough grading.

Vertical bearing and lateral resistance values indicated below are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by one-third percent for short duration loading, which includes the effect of wind or seismic forces.

10.2 Foundations

Residential one- or two-story wood-framed, lightly loaded building structures may be supported on conventional continuous and/or spread footings if the subgrades are properly prepared as described in the Section 8.0, *Earthwork/Site Grading Recommendations.*

The proposed structures may be supported on conventional continuous (strip) and/or isolated (spread) footings. Footings should be placed on compacted fill, the thickness of which should be equal to the maximum footing dimension or five (5) feet, whichever is greater.

Continuous and isolated shallow spread footings should be at least 24 inches wide and embedded at least 24 inches below lowest adjacent soil grade.

Footings placed at a depth of 24 inches below lowest adjacent grade may be designed based on an allowable net bearing capacity of 3,000 pounds per square foot (psf). The footings should have a minimum reinforcement equal to two #4 bars-one placed near the top and one placed near the bottom. Structural design may require wider footings and/or more reinforcement.

10.3 Retaining Walls

Cantilevered earth retaining walls should be designed based on an active earth pressure equal to that developed by an equivalent fluid density as shown in the following table. This pressure assumes a level ground surface behind the wall for a distance greater than the wall height. If water pressure is allowed to build-up behind the walls, the active pressure should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressure against the wall. At-rest earth pressure can be taken as equal to that developed by an equivalent fluid density as presented in the following table (d = 120pcf).

Gradient of Back Slope (horizontal:vertical)	Free Cantilever Wall Equivalent Fluid Weight (pcf)	Restrained Wall Equivalent Fluid Weight (pcf)
Flat	40	60

Resistance to lateral loads can assume to be provided by friction at the base of foundations and by passive earth pressure. A coefficient of friction 0.30 between concrete and soil may be used with the dead load forces. An ultimate passive earth pressure of 250 psf per foot of depth may be used for the sides of footings poured against recompacted native soil. The maximum value of the passive earth pressure should be limited to 2,500 psf. The lateral resistance provided by the friction and passive resistances may be combined directly without any reduction. These lateral resistances may be increased by 33 percent for seismic forces.

10.4 Settlement

Total settlements of footings placed on properly compacted fill or approved native soils should be 0.5-inch or less under the design loads assumed in this report. The differential settlement can be taken as equal to one half of the total settlement. For footings placed on bedrock, settlement can be considered to be negligible.

10.5 Slabs-on-Grade

The design of slabs-on-grade, among other factors, will depend on the expansive potential of the pad soils. Lot specific recommendations regarding the type of slabs-on-grade should be based on testing of the expansion index of the pad subgrade soils at the completion of grading. If slabs are placed on compacted fill, it is likely (based on medium expansion potential) that post-tension slabs will be required. Care should be taken to avoid slab curling if slabs are poured in hot weather. Conventional slabs-on-grade should be provided with a minimum of #3 bars on 24-inch centers, or as recommended by the structural engineer. Moisture sensitive slab should be protected by 10-mil-thick polyethylene vapor barriers. The barrier should be underlain by two (2) inches of sand to above and below. The polyethylene vapor barrier is only effective if it is above the exterior grades.

Subgrade for slabs-on-grade should be firm and uniform. All slab subgrades (with very low expansion potential) should be moisture-conditioned to within three (3) percent higher than optimum prior to the placement of concrete. All loose or disturbed soils including under slab utility trench backfills should be recompacted prior to the placement of clean sand underneath the moisture barrier.

10.6 Preliminary Pavement Design

<u>Asphalt Concrete Pavement Design</u>

An R-value of 36 is used to determine preliminary pavement structural sections. Site soils should be substantially mixed during site grading and the R-values of the final subgrade soils will be used to determine the pavement sections.

Analyses were performed to evaluate design structural sections for asphalt concrete pavements corresponding to Traffic Indices (TIs) ranging from 5 to 8 and an R-value of 36. The analyses were based on Caltrans' design procedure for flexible structural pavement sections without the recommended safety factor of 0.20 feet, when evaluating the required Gravel Equivalent (GE) of the Asphalt Concrete (AC). The Riverside County Transportation Department minimums are applied. The results of our analyses are summarized in the following table.

Table No. 3, Recommended Pavement Sections

R-value	Traffic Index (TI)	Pavement Sections	
		Asphalt Concrete (inches)	Aggregate Base (inches)
36	5.0	3.0	5.0
	6.0	3.0	8.0
	7.0	4.0	8.0
	8.0	4.0	11.0
	Or 8.0	5.0	9.0

Prior to placement of base aggregate, at least the upper 12 inches of subgrade soils should be scarified, moisture-conditioned, if necessary, and recompacted to at least 95 percent relative compaction as defined by ASTM Standard D-1557 test method.

Pavement subgrade should be prepared in accordance with Section 301 of the Standard Specifications for Public Works Construction (SSPWC, 2003). The upper 12 inches of subgrade should be compacted to a relative compaction of at least 95 percent as per ASTM Standard D1557 test method.

Base materials should conform to Section 200-2.2, "Crushed Aggregate Base," of the current Standard Specifications for Public Works Construction (SSPWC), and should be placed in accordance per Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC, and should be placed in accordance with Section 302.5 of the SSPWC.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

11.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

This report has been prepared to aid in the evaluation of the site, to prepare site-grading recommendations, and to assist the civil and structural engineers in the design of the proposed commercial structures.

Recommendations presented herein, are based upon the assumption that adequate earthwork monitoring will be provided by Converse. Excavation bottoms should be observed by a Converse representative. Structural fill and backfill should be placed and compacted during continuous observation and testing by this office. Footing excavations should be observed by Converse prior to placement of steel and concrete so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials.

The commercial pads may be customized, including retaining walls, general re-grading, and modification to landscaping. Any of these modifications may adversely impact existing foundation conditions, adjacent slope stability, and/or adjacent pads. It is strongly recommended that proposed pad modifications should be reviewed by Converse or an experienced geotechnical engineer and certified engineering geologist.

All individual commercial pad developers, should be made aware of the need for geotechnical evaluation of proposed foundation, grading, irrigation, and/or landscaping modifications.

12.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with the generally accepted professional engineering and engineering geologic principles and practice within our profession in effect at this time in Southern California. Our conclusions and recommendations are based on the results of field and laboratory investigations, combined with an interpolation of subsurface conditions between and beyond exploration locations.

As the project evolves, Converse should be retained to provide continued consultation and construction monitoring, which should be considered an extension of geotechnical investigation services performed to date. We should review plans and specifications to verify that the recommendations presented herein have been appropriately interpreted, and that the design assumptions used in this report are valid. Where significant design changes occur, Converse may be required to augment or modify the recommendations presented herein. Subsurface conditions may differ in some locations from those encountered in the explorations, and may require additional analyses and, possibly, modified recommendations.

This report was written for Transcan Development, LLC, and only for their design team. We are not responsible for technical interpretations made by others of our exploratory information, which has not been described or documented in this report. Specific questions or interpretations concerning our findings and conclusions may require written clarification to avoid future misunderstandings.

13.0 REFERENCES

BLAKE, T., 2000, *FRISKSP, Version 4.0, A Computer Program for the Probabilistic Prediction of Peak Horizontal Acceleration from Digitized California Faults*, Computer Services and Software, Newbury Park, California.

BLAKE, T., 2000, *UBCSEIS, Version 1.03, A Computer Program for the Computation of Uniform Building Code Seismic Design Parameters*, Computer Services and Software, Newbury Park, California.

BOWLES, J. E., 1982, *Foundation Analysis and Design*, McGraw-Hill, Inc.

CALIFORNIA BUILDING CODE (CBC), 2001, International Conference of Building Officials.

CALIFORNIA DEPARTMENT OF TRANSPORTATION (1992), *Highway Design Manual*, fourth edition.

CALIFORNIA DIVISION OF MINES AND GEOLOGY, 1994, *Fault Activity Map of California and Adjacent Areas*, Map No. 6.

UNIFORM BUILDING CODE (UBC), 1997, International Conference of Building Officials.

UNITED STATES GEOLOGICAL SURVEY, 1967, Riverside East, California, Quadrangle, 7.5-Minute Series (Topographic), Scale 1:24,000, revised 1980.

APPENDIX A

EXPLORATION PROGRAM

APPENDIX A

EXPLORATION PROGRAM

Our field investigation included a site reconnaissance of the property and a subsurface exploration program consisting of drilling twenty (20) exploratory borings. During the site reconnaissance, the surface conditions were noted and the locations of the borings were determined. The boring locations were located by pacing or by rough measurements relative to existing topography and boundary features, and should be considered accurate only to the degree implied by the method used.

The borings were drilled with a truck mounted drill rig equipped with an 8-inches hollow stem auger. A Converse geologist continuously logged and classified the soils in the field by visual examination in accordance with the Unified Soil Classification System. The field descriptions have been modified where appropriate to reflect laboratory test results.

Relatively undisturbed and bulk samples of the subsurface soils were obtained at frequent intervals in the borings. The undisturbed samples were obtained with a California Modified Sampler (2.4-inch inside diameter and 3-inch outside diameter) lined with thin brass sample rings. The soil was retained in the brass rings (2.4 inches in diameter and one inch in height). The central portion of the sample was retained and carefully sealed in waterproof plastic containers for shipment to our laboratory. Bulk soil samples were collected in plastic bags and brought to or laboratory.

A key to soil symbols and terminology used in the boring logs is included as Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols.* For Boring Logs, see Drawings No. A-2 through A-21, *Log of Borings.*

APPENDIX B

LABORATORY TESTING PROGRAM

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LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical requirements of the project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

Moisture Content and Dry Density

Results of moisture content and dry density tests performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure of the *in situ* dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Expansion Index Tests

An expansion index test was performed on a representative, near-surface soil sample in accordance with the CBC Standard to evaluate the expansion potential of the site soils. Test results are presented in the table below.

Table No. B-1, Results of Expansion Index Tests

Boring No./Depth (feet)	Description	Expansion Index	Expansion Potential
BH-19/0-5'	Silty Sand (SM), fine- to coarse-grained, trace to little clay, brown	37	Low

Laboratory Maximum Density Test

A laboratory maximum density-moisture content relationship test was performed on a representative bulk sample. Tests were conducted in accordance with ASTM Standard D1557 laboratory procedure. The test results are presented on Drawing No. B-1, *Moisture-Density Relationship Results.*

Direct Shear Tests

Direct shear testing was performed on representative soil samples in soaked moisture conditions. Three individual samples, each contained in a brass sampler ring, were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The sample was then sheared at a constant strain rate of 0.01 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Peak strength was selected from the shear-stress horizontal deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawing No. B-2, *Direct Shear Test Results* and in the table below.

Table No. B-2, Summary of Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-9	5'	Silty Sand (SM)	34	350

R-value Tests

Two (2) representative soil samples were tested to evaluate the Resistance (R) value in accordance with the State of California Test Method 301-G. The test results are presented in the following table.

Table No. B-3, Summary of R-value Test Results

Sample Location/Depth	Soil Description	R-value
BH-1/0-5'	Silty Sand (SC), fine- to medium-grained, brown	40
BH-10/0-3'	Silty Sand (SM), fine- to medium-grained, brown	36

Collapse Test

Three (3) collapse tests were performed on representative samples to evaluate its collapse potential. The tests were performed in accordance with the ASTM Standard D5333 test method. Test results are presented in the following table.

Table No. B-4, Summary of Collapse Test Results

Sample Location/Depth	Percent Collapse
BH-9/5'	3.4
BH-12/5'	0.2
BH-14/5'	0.6

Soil Corrosivity

A representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by M. J. Schiff & Associates in Claremont, California. A summary of the test results is presented below with the test results attached at the end of this appendix.

Table No. B-5, Soil Corrosivity Test Results

Boring No./Depth	pH	Chloride (ppm)	Sulfate (ppm)	Min. Resistivity (saturated) (ohm-cm)
BH-6/0-4'	7.4	ND	ND	8,000

Note: ND = Not Detected

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.

APPENDIX C

RECOMMENDED EARTHWORK SPECIFICATIONS

APPENDIX C

RECOMMENDED EARTHWORK SPECIFICATIONS

C1.1 Scope of Work

The work includes all labor, supplies and construction equipment required to construct the building pads in a good, workmanlike manner, as shown on the drawings and herein specified. The major items of work covered in this section include the following:

- Site Inspection
- Authority of Geotechnical Engineer
- Site Clearing
- Excavations
- Preparation of Fill Areas
- Placement and Compaction of Fills
- Observation and Testing

C1.2 Site Inspection

1. The Contractor shall carefully examine the site and make all inspections necessary, in order to determine the full extent of the work required to make the completed work conform to the drawings and specifications. The Contractor shall satisfy himself as to the nature and location of the work, ground surface and the characteristics of equipment and facilities needed prior to and during prosecution of the work. The Contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications must be brought to the Owner's attention in order to clarify the exact nature of the work to be performed.
2. This *Geotechnical Investigation Report* may be used as a reference to the surface and subsurface conditions on this project. The information presented in this above referenced report is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the boring logs. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information shall not relieve the Contractor from performing such independent investigation and evaluation to

satisfy himself as to the nature of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

C1.3 Authority of the Geotechnical Engineer

1. The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.
2. As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material; and (c) have the authority to approve or reject soils proposed for use in building areas.
3. The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the contractor and (c) the matters of compensation.

C1.4 Site Clearing

1. Clearing and grubbing shall consist of the removal from building and pavement areas to be graded: all existing pavement, utilities, and vegetation.
2. Organic and inorganic materials resulting from the clearing and grubbing operations shall be hauled away from the areas to be graded.

C1.5 Excavations

1. Based on observations made during our field explorations, the surficial soils can be excavated with conventional earthwork equipment.

C1.6 Preparation of Fill Areas

1. All organic material, organic soils, incompetent alluvium, undocumented fill soils and debris should be removed from the proposed building areas.
2. After the required removals have been made, the exposed native earth materials shall be excavated to provide a zone of structural fill for the support of footings, slabs-on-grade, exterior flatwork and pavements. All loose, soft or disturbed earth materials should be removed from the bottom of excavations before placing structural fill. As a minimum, the on site soils in the building area and to five (5) feet beyond the building limits and appendages shall be removed and recompacted to provide at least five (5) feet of properly compacted fill underneath all slabs and three (3) feet of compacted fill underneath all footings.
3. The subgrade in all areas to receive fill shall be scarified to a minimum depth of six (6) inches, the soil moisture adjusted to at least two (2) percent above

optimum for fine-grained soils and within three (3) percent of optimum moisture content for granular soils, and then compacted to at least 90 percent of maximum dry density as determined by ASTM Standard D1557 test method. Scarification may be terminated on moderately hard to hard, cemented earth materials with the approval of the Geotechnical Engineer.

4. Compacted fill may be placed on native soils that have been properly scarified and recompacted as discussed above.

5. All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

C1.7 Placement and Compaction of Fill

1. Compacted fill placed for the support of footings, slabs-on-grade, exterior concrete flatwork, driveway and parking pavements will be considered structural fill. Structural fill may consist of approved onsite soils or imported fill that meets the criteria indicated below.

2. Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer shall be placed in layers on approved earth materials. Soils used as compacted structural fill shall have the following characteristics:

   a. All fill soil particles shall not exceed three (3) inches in nominal size, and shall be free of organic matter and miscellaneous inorganic debris and inert rubble.

   b. In order to limit moisture penetration to foundation earth materials, imported fill materials shall be similar to on-site earth materials with at least 30 percent passing the No. 200 sieve. As an alternative to 30 percent passing the No. 200 sieve, import materials with a remolded permeability of 1×10^{-6} cm/sec or less would be acceptable.

   c. Imported fill materials shall have an Expansion Index (EI) less than 20. All imported fill should be compacted to at least 90 percent of maximum dry density (ASTM D1557) at about two (2) percent above optimum moisture for fine-grained soils, and within three (3) percent of optimum for granular soils.

   d. Imported fill materials shall have less than 0.1 percent sulfate salts, if possible. If laboratory test results indicate import fill materials contain more than 0.1 percent sulfate salts, a concrete mix should be designed to resist the sulfate levels indicated by the laboratory test results.

3. Fill soils shall be evenly spread in maximum eight-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

4. All fill placed at the site shall be compacted to at least 90 percent of the maximum laboratory density as determined by ASTM Standard D1557 test method. Granular soils shall be moisture conditioned to within three (3) percent, and fine-grained soils to at least two (2) percent above, optimum moisture content.

5. Fill exceeding five (5) feet in height shall not be placed on native slopes that are steeper than 5 to 1 (horizontal to vertical). Where native slopes are steeper than 5 to 1, and the height of the fill is greater than five (5) feet, the fill shall be benched into competent materials. The height and width of the benches shall be at least two (2) feet.

6. Representative samples of materials being used as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 compaction method.

7. Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations shall not resume until the Geotechnical Engineer approves the moisture and density conditions of the previously placed fill.

8. It shall be the Grading Contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with job specifications, all berms have been properly constructed, and all associated drainage devices meet the requirements of the Civil Engineer.

C1.8 Observation and Testing

1. During the progress of grading, the Geotechnical Engineer will provide observation of the fill placement operations.

2. Field density tests will be made during grading to provide an opinion on the degree of compaction being obtained by the contractor. Where compaction of less than specified herein is indicated, additional compactive effort with adjustment of the moisture content shall be made as necessary until the required degree of compaction is obtained.

3. A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.